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Shaking table 2-D models of a concrete gravity dam†

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SUMMARY

One of the most famous and studied cases of dams subjected to earthquake loading is the Koyna Dam in India. In this study, a two-dimensional model of Koyna Dam at $\frac{1}{50}$ scale was used on a shake table to simulate effects and serve as data for non-linear computer model calibration. A new concrete mix was designed for the non-linear similitude modelling. This new mix provided the correct kinematic failure of concrete at scale. Two models were tested to failure: one with an initial shrinkage crack and one monolith. Reservoir effects were not modelled. The results of both models are discussed and compared. The ability to model non-linear effects is discussed. Published in 2000 by John Wiley & Sons, Ltd.

KEY WORDS: concrete earthquake Koyna modelling dams

INTRODUCTION

One of the most famous and studied cases of dams subjected to earthquake loading is the Koyna Dam in India. This 338 foot (103 m) high concrete gravity dam suffered cracking during a magnitude 6.5 earthquake in 1967 [1]. During this earthquake, the ground acceleration in the stream direction reached 0.49 g, with a total duration of strong shaking lasting about 4 s. At the time of the event the reservoir was 37 ft (11 m) below the crest.

Following the Northridge Earthquake on 17 January 1994 and the earthquake in Kobe, Japan one year later on 17 January 1995, new information about vertical acceleration magnitudes was available. Continuing concerns about the performance of concrete dams subjected to severe earthquakes has stimulated research to find new approaches to analyse and predict this performance using non-linear numerical analysis techniques [2]. In some cases, linear dynamic analyses indicate high stresses which can only be studied using non-linear models.

Several studies have been conducted on gravity dam monoliths [2–5]. In References [2, 3], attention was given to developing a modelling material which maintained similitude with the

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prototype. In Reference [2], test results were compared to linear elastic analysis results. More recent studies have been completed using models tested in centrifuges [6, 7]. This more recent work was developed to provide data which can be used for comparison to numerical models.

The purpose of this investigation was to produce results that could be compared to non-linear computer models. The geometry of the model was scaled from the Koyna Dam and followed previous work [2, 3]. The models were designed, to the extent possible, to maintain similitude relationships and yet be simple enough for direct comparison with computer-predicted results. To this end, unlike the previous studies [2, 3], similitude with reservoir effects is not attempted thereby eliminating the need to model coupling effects. Two models were tested, a model with a natural pre-existing crack and a monolithic model failed during testing.

EXPERIMENT SET-UP AND PROCEDURE

The scale chosen for this model was a $\frac{1}{50}$ geometric scale. Similitude requirements for models have been summarized in other references [8] and estimated properties of Koyna Dam have also been suggested [2]. These properties are summarized in Table I.

Concrete mix design and material properties

For this study, a new low strength concrete mix was designed. Considerable work had been accomplished in previous studies [2, 3, 9] to produce an appropriate similitude concrete mix. As has been suggested, curing and the associated shrinkage cracking can be problematic when using concrete mixes having highly reduced properties. In addition, the use of any lead product to meet density requirements needs to be analysed to ensure that requirements for handling, storage, and disposal of hazardous wastes are met. This latter problem, in particular, limits the ability to have the material commercially produced and complicates the disposal of such materials. In addition, when modelling non-linear failure, consideration must be given to reproducing the correct failure mechanism at model scale.

The concrete mix for this study used bentonite pellets as a component to reduce strength. The use of bentonite pellets poses a problem logistically since saturation of the pellets is required prior to mixing. The mix components for the trial laboratory-mixed concrete and the commercially mixed model concrete are shown in Table II.

This trial mix was initially made in the laboratory with bentonite hydration accomplished overnight. Based on the apparent success of this mix, both shake table models were made using

Table I. Estimated concrete properties, the associated scale factors, and the model material target values.

Property	Prototype estimate	Scale factor	Target value
E	27 940 000 kN/m^2 (4,000,000 lb/in^2)	50	558 800 kN/m^2 (80,000 lb/in^2)
f_c'	27 940 kN/m^2 (4,000 lb/in^2)	50	558.8 kN/m^2 (80 lb/in^2)
f_t	2794 kN/m^2 (400 lb/in^2)	50	55.9 kN/m^2 (8 lb/in^2)
Density	2403 kg/m^3 (150 lb/ft^3)	1	2403 kg/m^3 (150 lb/ft^3)
ε_u^c	0.0025	1	0.0025
ε_u^t	0.00012	1	0.00012

Table II. Model concrete mix components.

	Design quantities per yd^3 ($0.765\ m^3$) (laboratory mix)		Yield quantities per yd^3 ($0.765\ m^3$) (actual model mix)	
Component	Lab mix	Volume in mix per $0.765\ m^3$ batch	Model mix	Volume in mix per $0.765\ m^3$ batch
Air		$0.0040\ m^3$ ($0.14\ ft^3/yd^3$) (assumed by $\frac{1}{2}$% entrapped air)		$0.0147\ m^3$ ($0.52\ ft^3/yd^3$)
Water	$332\ kg/m^3$ ($560\ lb/yd^3$)	$0.2546\ m^3$ ($8.99\ ft^3/yd^3$)	$285\ kg/m^3$ $480\ lb/yd^3$	$0.2175\ m^3$ ($7.68\ ft^3/yd^3$)
Cement	$95\ kg/m^3$ ($160\ lb/yd^3$)	$0.0232\ m^3$ ($0.82\ ft^3/yd^3$)	$100\ kg/m^3$ $168\ lb/yd^3$	$0.0244\ m^3$ ($0.86\ ft^3/yd^3$)
Bentonite	$24\ kg/m^3$ ($40\ lb/yd^3$)	$0.0071\ m^3$ ($0.25\ ft^3/yd^3$)	$25\ kg/m^3$ $42\ lb/yd^3$	$0.0074\ m^3$ ($0.26\ ft^3/yd^3$)
Sand	$810\ kg/m^3$ ($1{,}366\ lb/yd^3$)	$0.2379\ m^3$ ($8.4\ ft^3/yd^3$)	$863\ kg/m^3$ $1454\ lb/yd^3$	$0.2512\ m^3$ ($8.87\ ft^3/yd^3$)
No. 4–3/8″ Gravel	$328\ kg/m^3$ ($553\ lb/yd^3$)	$0.0951\ m^3$ ($3.36\ ft^3/yd^3$)		
3/8″–3/4″ Gravel	$492\ kg/m^3$ ($829\ lb/yd^3$)	$0.1427\ m^3$ ($5.04\ ft^3/yd^3$)	$865\ kg/m^3$ $1458\ lb/yd^3$	$0.2495\ m^3$ ($8.81\ ft^3/yd^3$)

Note: $w/c = 3.5$, where w/c = water-to-cement ratio by mass.
$B/(B + C) = 0.2$ by mass, where B = bentonite mass and C = cement mass.

this bentonite–concrete mix design. Owing to the volume required for the shake table models (6 cubic yards including test cylinders), the actual model mix was ordered and supplied commercially. For the commercial mix, hydration was attempted in the mixer drum during transit. At the batch plant the water was reduced from the original design to decrease sloshing in transit. On-site water was added to achieve a slump of approximately 7.5 in which was believed would indicate a mix similar to the laboratory mix. The resulting water content for the model mix was lower than the original laboratory mix due to incomplete hydration of the bentonite during transit. The incomplete hydration of bentonite resulted in a higher free-moisture content, and thus higher slump for a given water content. Table III shows the properties of each of the three mixes. Comparing the properties of each mix shows that slump is not a good predictor of the cured strength.

Laboratory testing was done in support of each experiment. Standard $6'' \times 12''$ cylinders of the bentonite concrete were made from each batch. Stress–strain data for a typical compression test is shown in Figure 1. Of particular significance, typical of normal concrete, breaks for all compressive cylinder tests failed in a classic shear plane of approximately 65°. Other trial mixes were tested in the lab based on lead and plaster combinations and these materials created failure modes such as horizontal layer crushing which is not characteristic of concrete. The bentonite concrete modelled the kinematic failure mechanism better than the materials made from a combination of plaster and lead; however, it is clear that not all parameters matched the similitude requirements

Table III. Properties of modal materials.

Property	Target value	Laboratory results	Actual Koyna I Mix	Actual Koyna II mix
Slump	200 mm (8 in)	200 mm (8 in)	190 mm (7.5 in)	200 mm (8 in)
Density	2400 kg/m^3 (150 lb/ft^3)	2132 kg/m^3 (133.1 lb/ft^3	2162 kg/m^3 (135 lb/ft^3)	2211 kg/m^3 (138 lb/ft^3)
Static modulus of elasticity				
7 days	—	—	290 000 kPa (42 000 lb/in^2)	—
15 days	—	—	—	1 082 000 kPa (157 000 lb/in^2)
28 days	558 800 kPa (80 000 lb/in^2)	510 000 kPa (74 000 lb/in^2)	379 000 kPa (55 000 lb/in^2)	—
Dynamic modulus of elasticity				
15 days	—	—	—	779 000 kPa (113 000 lb/in^2)
28 days	—	641 000 kPa (93 000 lb/in^2)	—	—
35 days	—	—	552 000 kPa (80 000 lb/in^2)	—
Ultimate static compressive strength				
7 days	—	345 kPa (50 lb/in^2)	614 kPa (89 lb/in^2)	—
15 days	—	—	—	1400 kPa (203 lb/in^2)
28 days	560 kPa (80 lb/in^2)	579 kPa (84 lb/in^2)	1062 kPa (154 lb/in^2)	—
120 days	—	—	2000 kPa (290 lb/in^2)	—
Static tension				
Splitting Tension @ 15 days	—	—	—	186 kPa (27 lb/in^2)
Beam Tension @ 15 days	—	—	—	414 kPa (60 lb/in^2)
Direct Tension @ 21 days	—	—	97 kPa (14 lb/in^2)	—
Beam Tension @ 21 days	—	—	221 kPa (32 lb/in^2)	—
Splitting Tension @ 28 days	56 kPa (8 lb/in^2)	83 kPa (12 lb/in^2)	138 kPa (20 lb/in^2)	—
Beam Tension @ 28 days	56 kPa (8 lb/in^2)	—	338 kPa (49 lb/in^2)	—
Dynamic tension				
Splitting Tension @ 15 days	—	—	—	359 kPa (52 lb/in^2)
Splitting Tension @ 28 days	—	152 kPa (22 lb/in^2)	—	—
Ultimate strain				
ε_u^c	0.0025	0.004	0.005	0.004

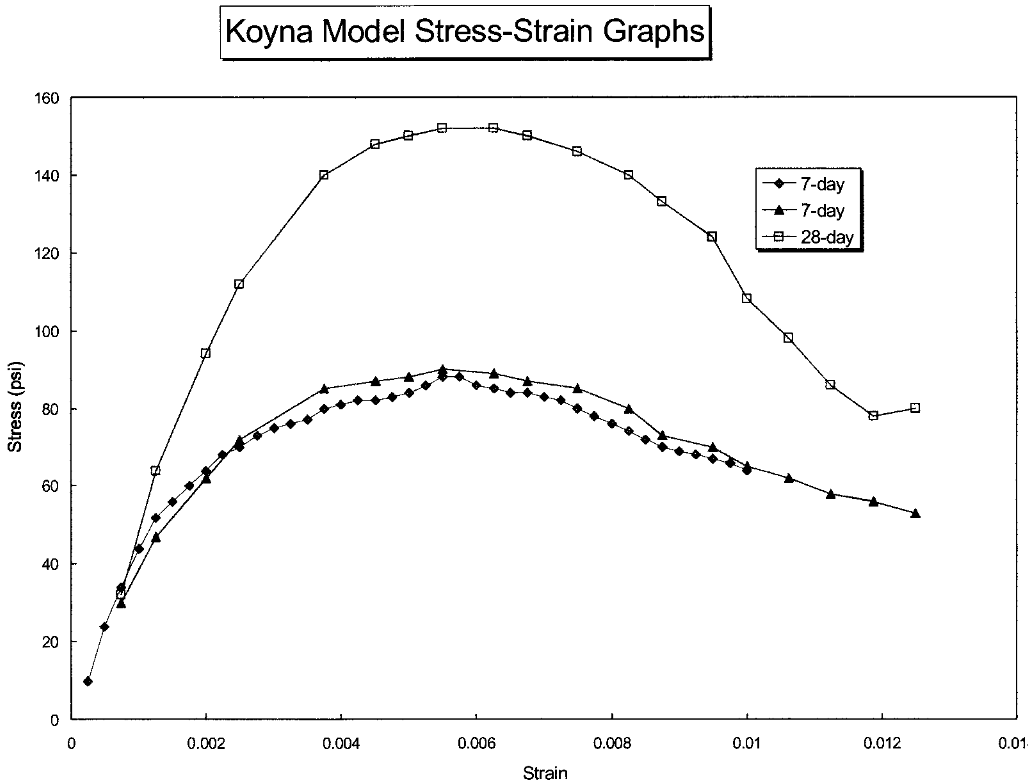

Figure 1. Stress–strain graphs.

simultaneously. Changes in mix water had the largest effect on the elastic properties. As was stated previously, the primary intent of this test program was to produce calibration data for verification of computer models and it is believed that more accurately modelling the kinematic failure mechanism with a material that compares favourably (in the ballpark) in similitude relationships is a significant contribution toward this end.

In addition to the standard suite of laboratory tests, specialized tests were run to measure properties typical of non-linear computer material models. Typical fracture data (crack width vs. load in beam tension) is shown in Figure 2. The beams used were ASTM standard—$6'' \times 6'' \times 21''$ beams with a $1''$ notch in the beam centre. Figure 3 shows unload–reload data typical for plasticity models. These tests were not intended to provide an exhaustive set of material properties for all published numerical models, but the properties measured are believed to be representative.

Model construction and instrumentation

Tests were completed in the U.S. Bureau of Reclamation's Materials Engineering and Research Laboratory. The Vibration Laboratory for large-scale tests has been in existence at Reclamation

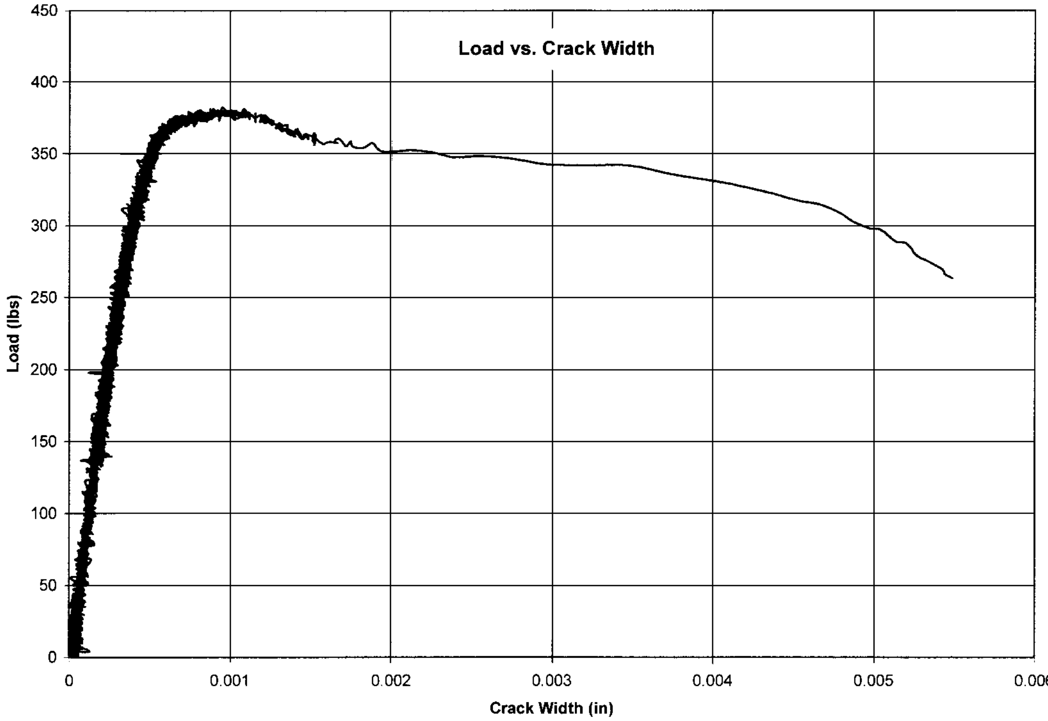

Figure 2. Load vs. crack width in beam tension.

since 1969 [10]. For these experiments the models were constructed on a shake table and excited in a single axis corresponding to a horizontal motion along the upstream–downstream axis. The table's response was characterized using modal analysis and tested in motion to determine the system's upper frequency response limit. The table's lowest natural frequency was measured at 30-Hz. The response was acceptable at frequencies below 26-Hz. Because of this upper frequency limitation, a similitude simulation of the motion of an earthquake was not used. A sinusoidal excitation was selected for practical reasons associated with the table, and for simplicity in numerical model calibration.

The second model is shown after testing in Figure 4, the first Koyna model is shown mounted on the shake table in Figure 5. The $\frac{1}{50}$ scale chosen resulted in a 8.5 ft (2.6 m) tall model weighing 7850 lb. A slab representing the foundation was poured monolithically with the model to provide a fixed lower boundary at the base of the dam. All-thread rods were imbedded in the foundation to provide a means of anchoring the model to the shake table. Instrumentation was designed to measure displacements and accelerations on the model and from the input actuator. The general instrumentation locations are shown in Figure 6 and detailed in Table IV.

The first model was cast laying down on its side adjacent to the shake table. In this position forming and placing was much easier having an entire face for access and only a 1 ft 9 in depth of material. After a period of approximately 20 days, a small shrinkage crack appeared on the exposed face. At this time tension tests were run which may be useful in modelling the onset of

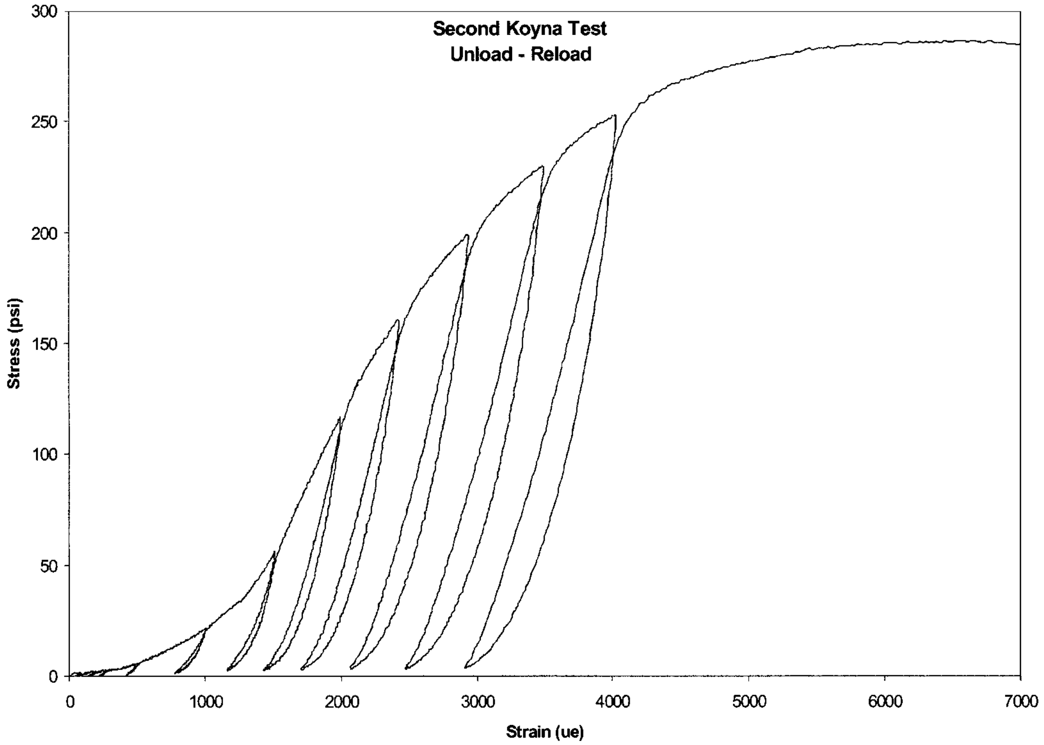

Figure 3. Unload–reload test showing plastic behaviour of the low-strength concrete.

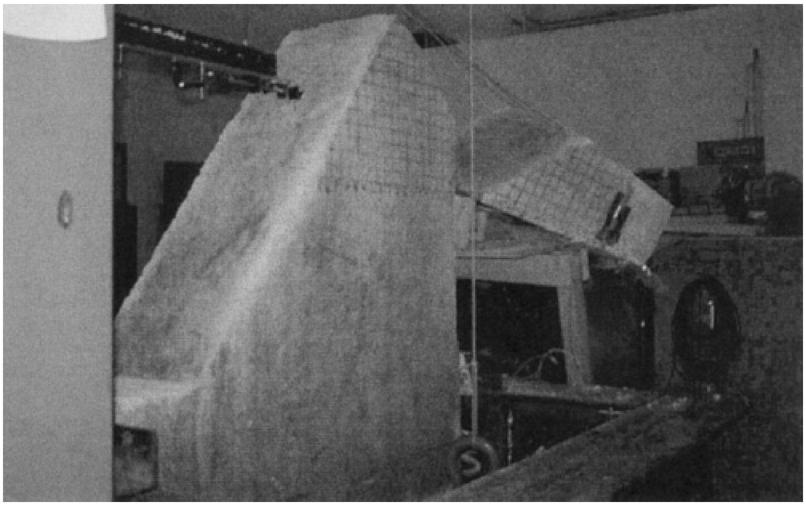

Figure 4. Second Koyna model failure plane.

Figure 5. First Koyna model mounted on the shake table. The shrinkage crack and eventual failure plane is sketched in.

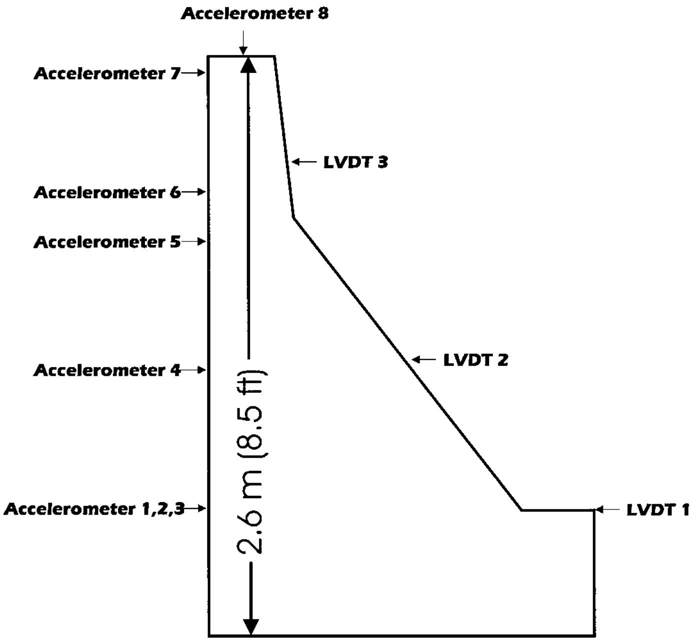

Figure 6. Instrument locations.

Table IV. Instrument locations.

Instrument ID	Type	Orientation	Height from base
Accelerometer 1	Acceleration	Horizontal, x-direction	0
Accelerometer 2	Acceleration	Horizontal, y-direction	0
Accelerometer 3	Acceleration	Vertical	0
Accelerometer 4	Acceleration	Horizontal, x-direction	0.66 m (2.17 ft)
Accelerometer 5	Acceleration	Horizontal, x-direction	1.22 m (4.00 ft)
Accelerometer 6	Acceleration	Horizontal, x-direction	1.47 m (4.83 ft)
Accelerometer 7	Acceleration	Horizontal, x-direction	2.03 m (6.67 ft)
Accelerometer 8	Acceleration	Vertical	2.03 m (6.67 ft)
LVDT 1	Displacement	Horizontal, x-direction	0
LVDT 2	Displacement	Horizontal, x-direction	0.97 m (3.17 ft)
LVDT 3	Displacement	Horizontal, x-direction	1.69 m (5.54 ft)

shrinkage. At approximately 28 days, the model was lifted onto the shake table and the forms were removed. The shrinkage crack was evident on the side of the model and on the sloped face and was assumed to extend through the model to the other two adjacent faces. The plane of the crack had an inclination of approximately 20° degrees from horizontal towards the side of the model. After approximately 1 additional week, the surface had dried sufficiently to apply instrumentation and the test was run.

The second model was cast upright in the shake table and was tested at a 15 day age to avoid the shrinkage cracking experienced in the first model. By casting upright, and testing earlier, the onset of shrinkage cracking was avoided and the second model produced a material failure under dynamic loading. Another benefit of testing the model earlier was the lower strength of the material. A complete suite of laboratory tests were performed on the material immediately following the testing of the model.

Input motions

Numerical analysis predicted that the fundamental model of the model was approximately 14 Hz; however, this predicted fundamental mode was out of plane with the direction of excitation for the test, being side to side in the model. The cantilever mode, mode 2 for the model but the first mode in plane with the excitation, was predicted to be at approximately 28 Hz. Model response was recorded at even frequencies from 2 to 28 Hz with a constant input acceleration of $0.1g$ to determine resonant frequencies of the modal. Figure 7 shows the acceleration of the top of the model along the excitation axis at even frequencies from 10 to 28 Hz. The first excitation frequency which showed an amplification of acceleration above the input was 14 Hz. Although the predicted modal response for this frequency is out of plane, the effect was demonstrated in the plane of testing. Higher frequencies did produce a more dramatic effect. A sinusoidal motion of 14 Hz (approximately 2 Hz prototype) was chosen as the excitation frequency for all subsequent tests. This lowest resonant frequency was believed to be the easiest for numerical simulation and calibration. The seismic record for upstream/downstream motion for the Koyna event, see Figure 8, has a primary component at 2.4 Hz. This is more readily seen in the response spectrum of Figure 9. A 2-Hz frequency for the Koyna event scales to approximately 14 Hz for the models.

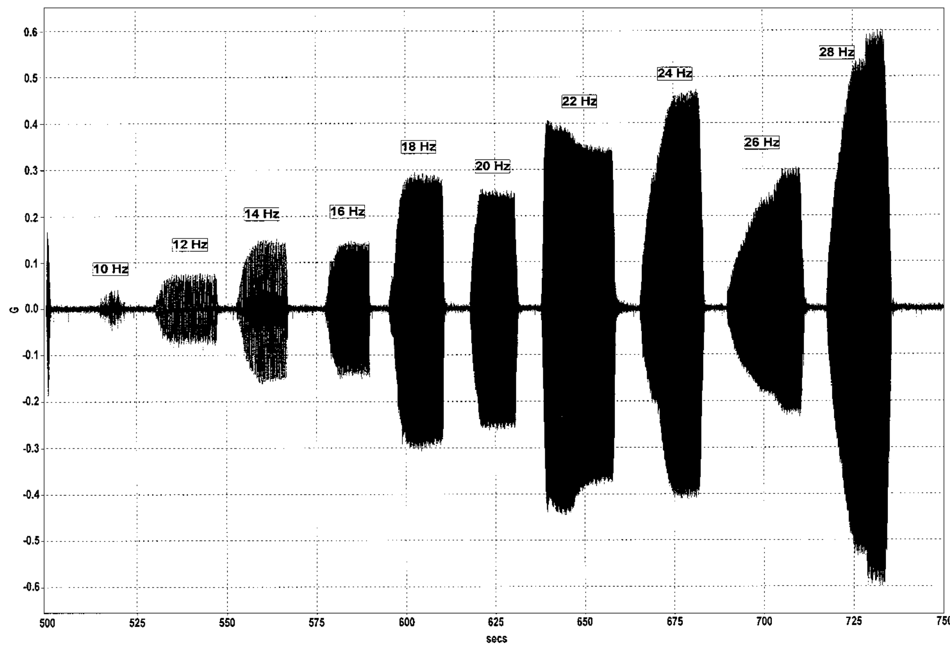

Figure 7. First Koyna test: horizontal acceleration at the top of the model.

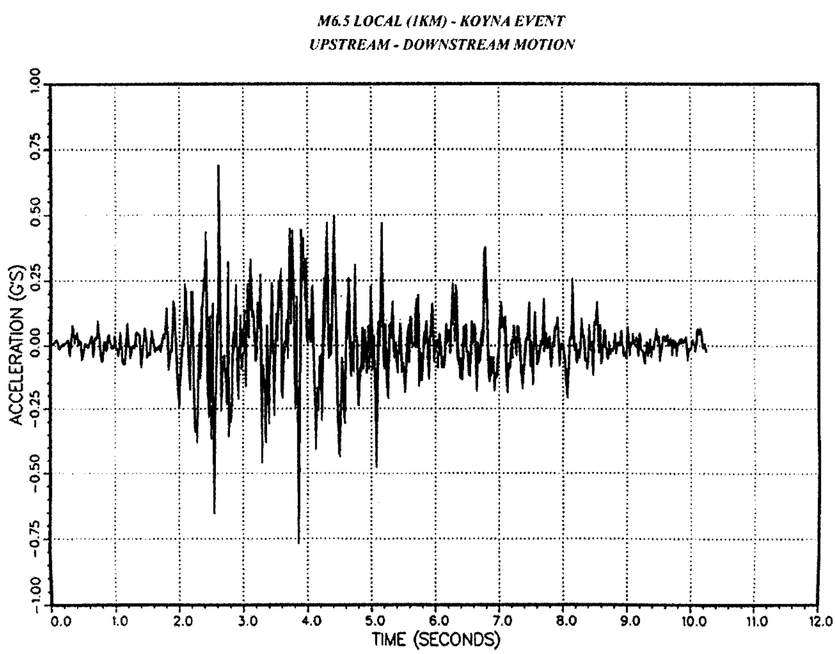

Figure 8. The seismic record for upstream/downstream motion during the Koyna event.

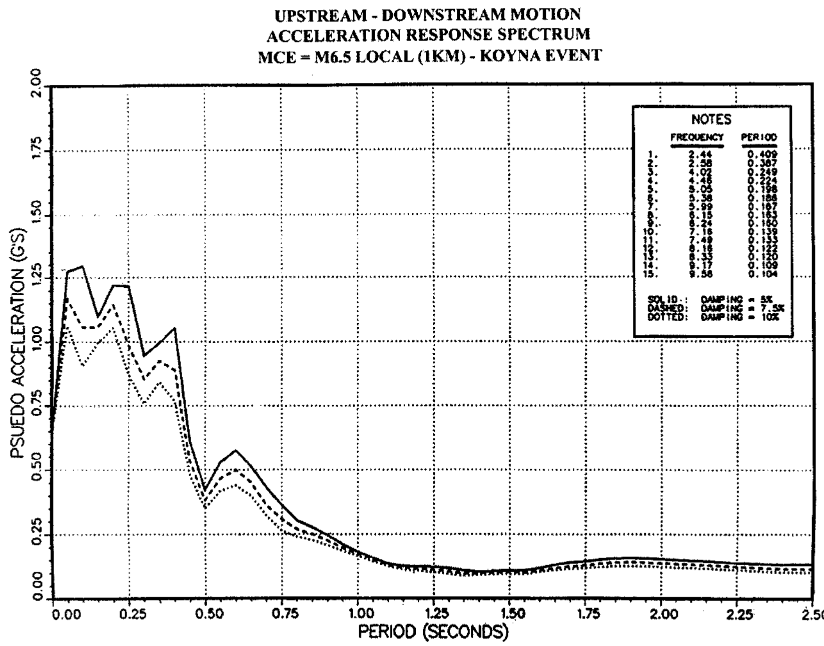

Figure 9. Koyna response spectrum.

Using this set frequency, the acceleration in the upstream/downstream direction of the model was increased until failure occurred.

To summarize, the test program for each model consisted of the following two phases: (1) Determine the lowest resonant frequency by shaking the model at $0.1g$ at even frequencies from 2 to 28 Hz, and (2). Fail the model by shaking it at the lowest resonant frequency, increasing the acceleration amplitude from $0.25g$ to failure in $0.25g$ steps holding at each step for 30 s.

TEST RESULTS

Model 1—cracked model

Four typical acceleration plots are shown in Figures 10–13. Figure 10 is of particular interest. This figure shows the acceleration at the base of the dam and at the base of the known crack to have a magnitude of $0.5g$, while at the crest the acceleration is nearly $2g$. This amplification of acceleration by a factor of 4 from the base to the top of the model, is similar to previous tests [2, 11]. The model displayed no failure characteristics at this acceleration which corresponds to observations made in the field during the Koyna earthquake. This acceleration amplification is attributed to the cantilever mode of vibration of the model. Numerical comparisons have been completed for this case [12].

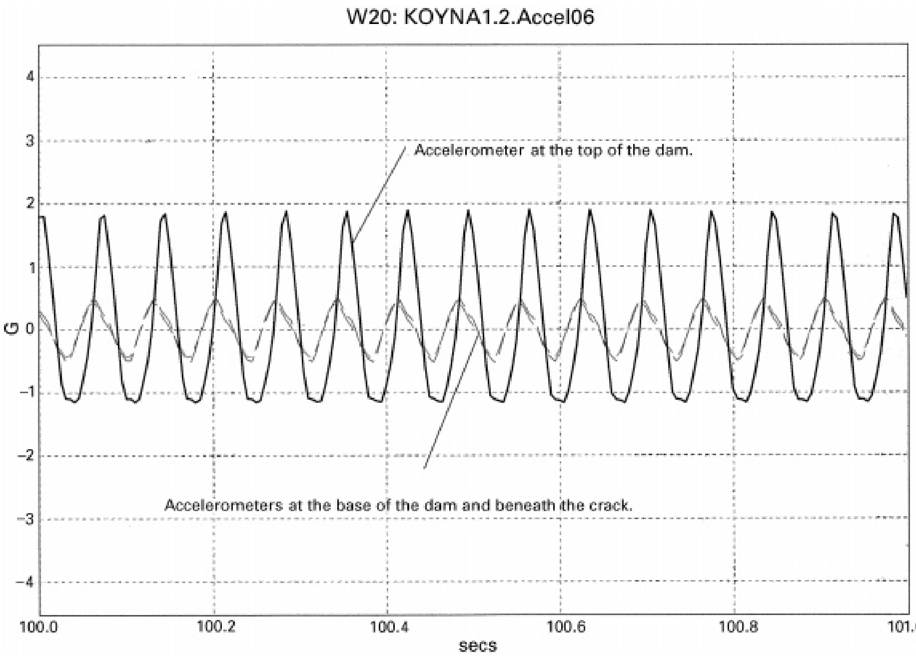

Figure 10. First Koyna model: base acceleration of 0.5g.

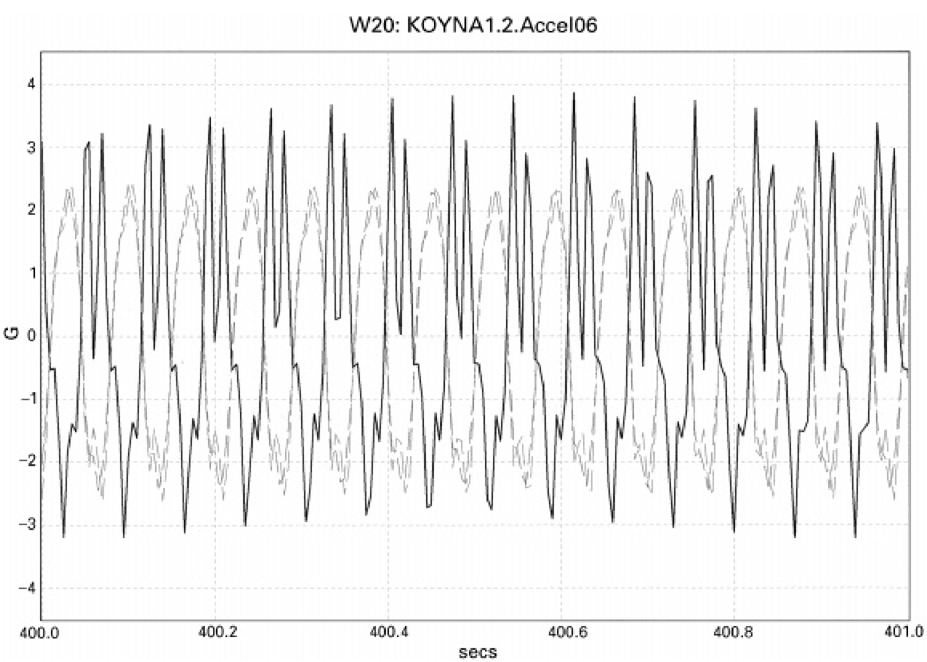

Figure 11. First Koyna model: base acceleration of 2.25g.

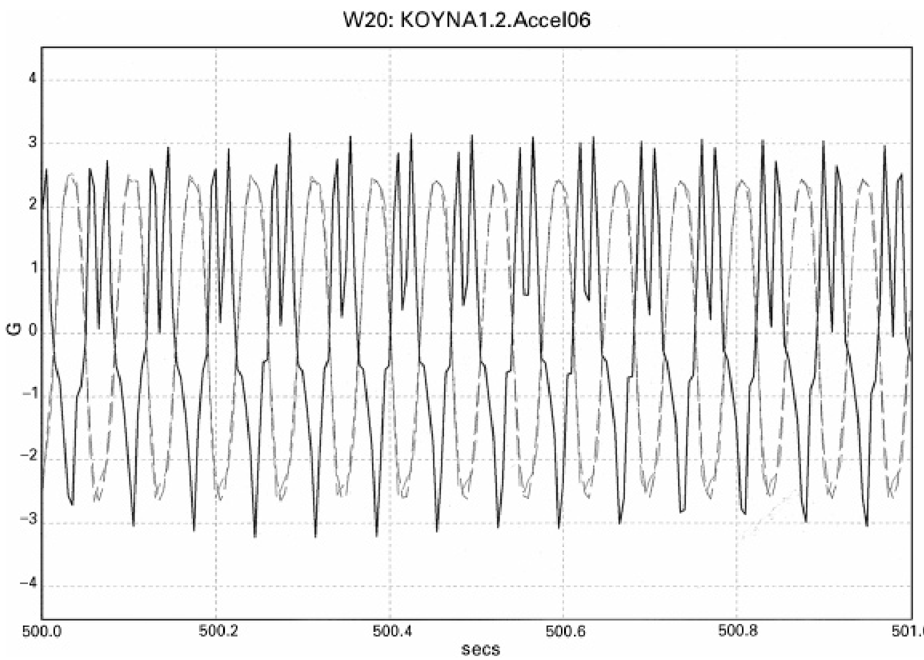

Figure 12. First Koyna model: base acceleration of 2.5g.

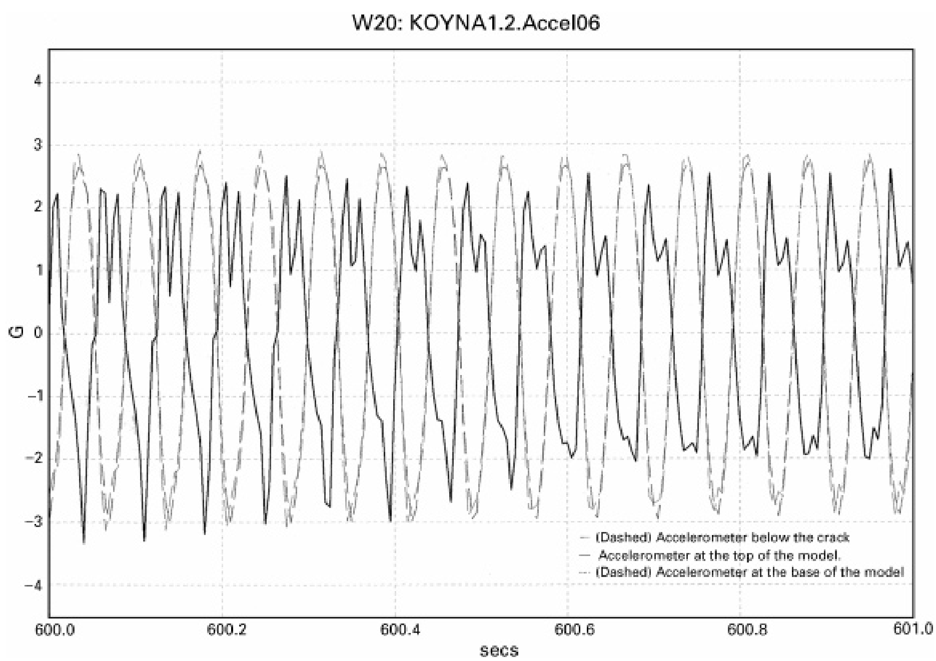

Figure 13. First Koyna model: base acceleration of 2.75g.

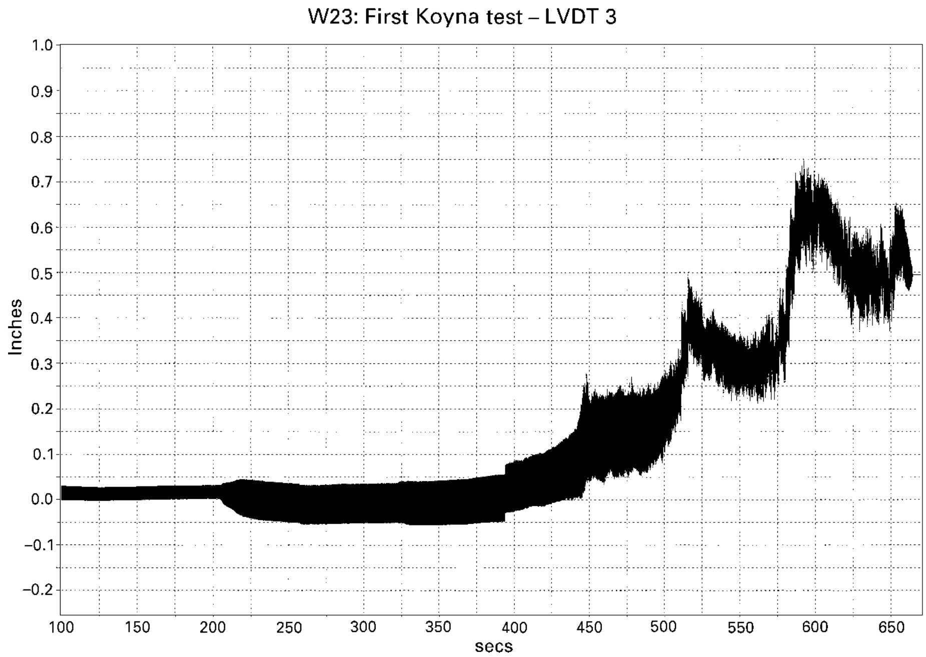

Figure 14. First Koyna model: displacement at the top of the model.

At a base acceleration of about $2g$, the top of the dam model began to show a puffing of material from the crack. This was caused by a rocking effect of the top acting as a bellows and blowing worn material from the cracked surface.

The next increment in acceleration, $2.25g$, showed a change in response of the portion of the model above the crack. As can be seen in Figure 11, the amplification of acceleration from base to top of the model decreases to a factor of 1.6. This compares to a factor of 4 previously. There is also an evident phase shift between the acceleration of the top and bottom at this point in the test.

As $2.5g$ the acceleration at the top and bottom of the model are nearly equal and 180 degrees out of phase as can be seen in Figure 12. Figure 14 shows displacement of the top of the model that indicates that it is sliding down the failure plane at this time.

At a base acceleration of $2.75g$, the bottom motion is at a higher acceleration than the top of the dam, Figure 13. By this point in the test the displacement of surfaces is well under way and the base motion is not readily transferred to the top section. The slippage is approximately $1/2''$ as shown in Figure 14. The cross-section maintained stability; sliding progressed slowly during the test. The top block could be observed to be progressively sliding down the preexisting shrinkage crack surface.

Model 2—monolithic model

As with the first model, a frequency sweep was completed first. Acceleration, normalized to the base motion, are shown in Figure 15. In comparing the response with that of the first model,

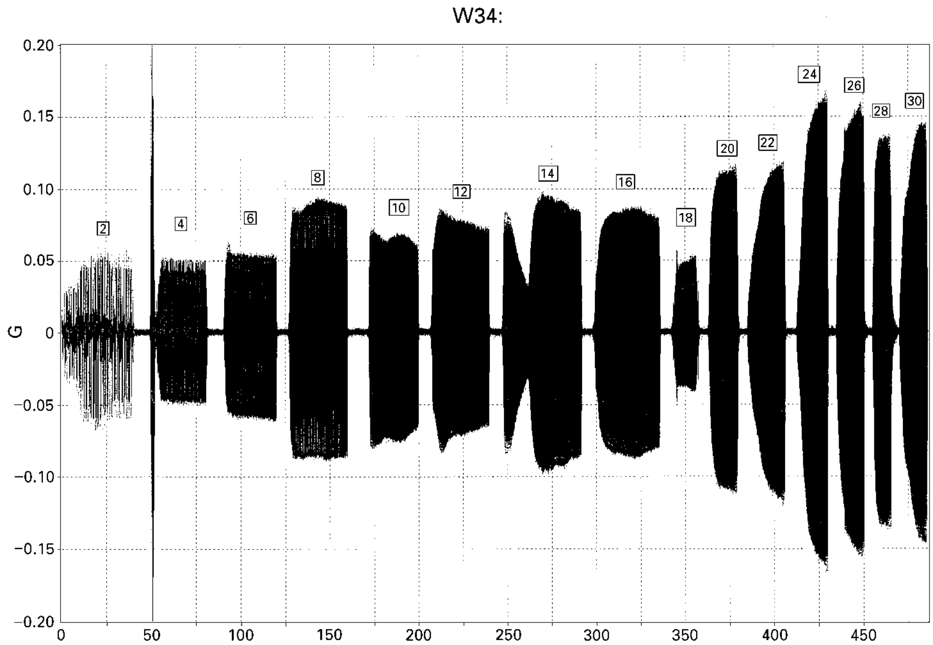

Figure 15. Second Koyna model: frequency sweeps.

Figure 7, some differences are noticed in the response characteristics. In Model 2, the response at 20 Hz seemed to indicate a fundamental mode. In both models 24 Hz seemed to indicate the first cantilever mode. These differences are believed to be inherent differences in the two models as built, but generally the two models appear similar in their modal response.

This model was tested to failure using the same procedure as the first model, with the total test period being almost 8 min and failure occurring at a base acceleration of 2.2g. The top of the dam did topple from the model, and the angle of failure was consistent with previous studies [11].

The model was videotaped during testing. Review of the tape revealed that the crack was not visible in one frame and had propagated through the structure by the next video frame. Standard video frame rates are $\frac{1}{30}$ of a second indicating that the crack developed in less than 0.03 s.

Analysis of the test data revealed anomalous behavior of the structure beginning approximately 330 s into the test (Figures 16–19). This behaviour is most prominently displayed in Figure 18, which is the vertical acceleration of the model as measured at the top of the structure. It can be seen that up to the 330 s point in the test, the vertical acceleration increases linearly with increasing horizontal input acceleration. This response is as expected and is attributed to a slight flexing of the shake table frame. At around the 330 s time frame the vertical acceleration starts increasing dramatically and continues to increase throughout the duration of the test. This increase is accompanied by a corresponding decrease in the horizontal acceleration of the top of the structure as seen in Figure 17. The displacement of the top of the model is shown in Figure 19 and displays a rather abrupt decrease in the displacement of the top which would correspond with the decreased acceleration. These phenomena are not believed to be related of the failure of

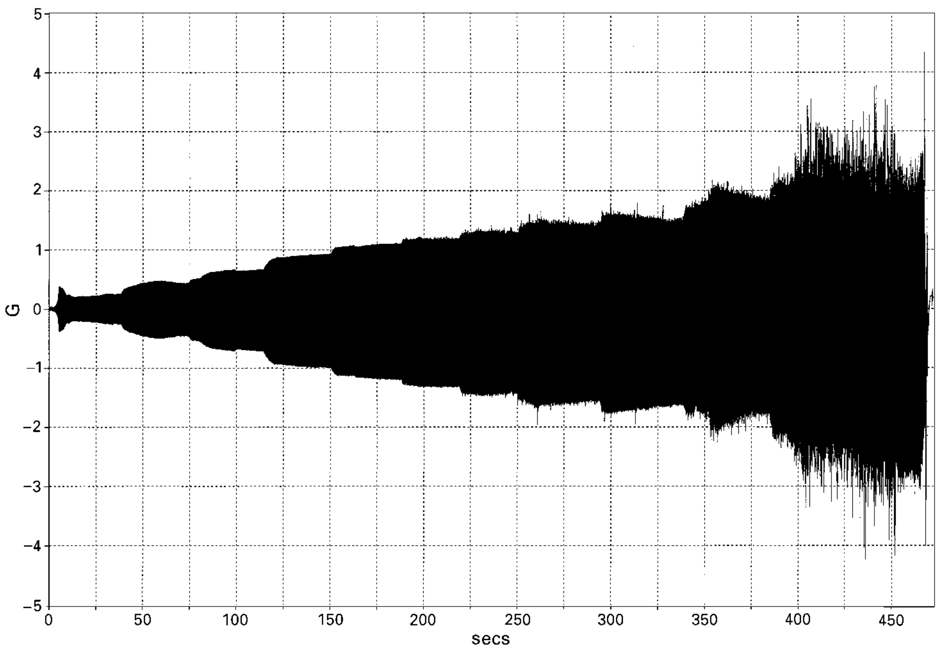

Figure 16. Second Koyna model: horizontal acceleration at the base of the model.

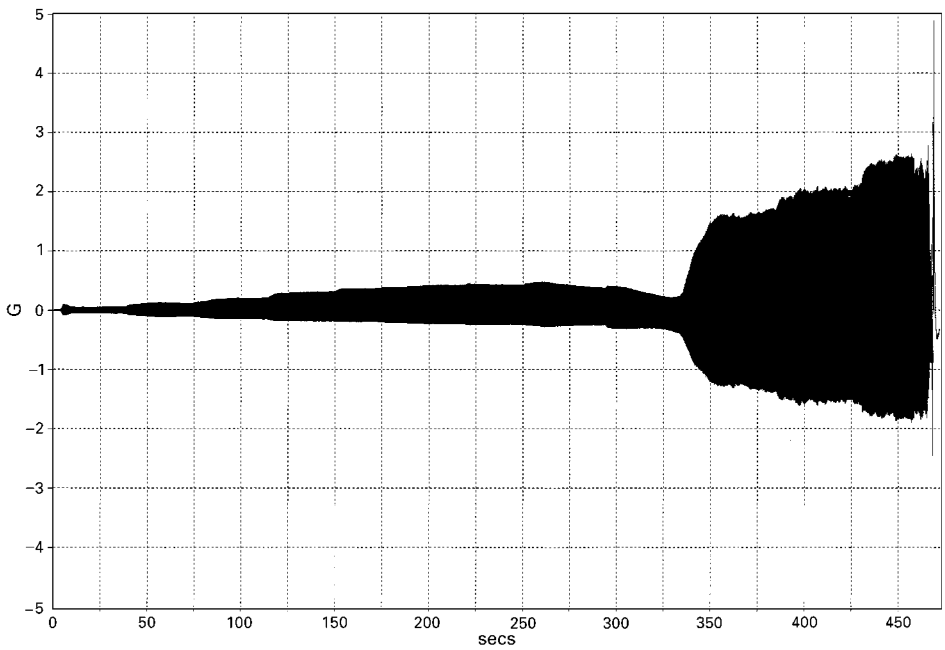

Figure 17. Second Koyna model: horizontal acceleration at the top of the model.

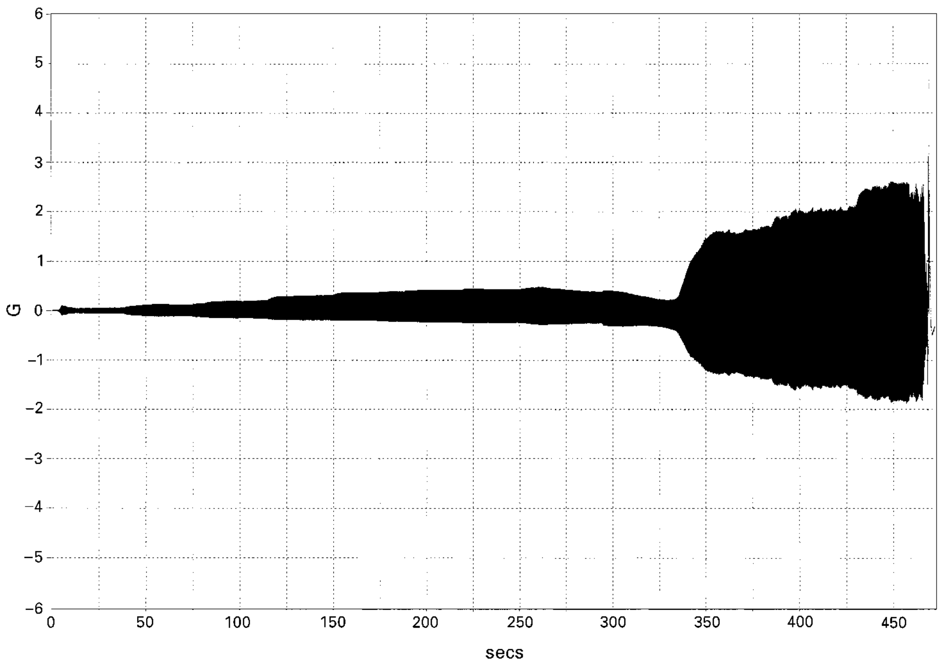

Figure 18. Second Koyna model: vertical acceleration at the top of the model.

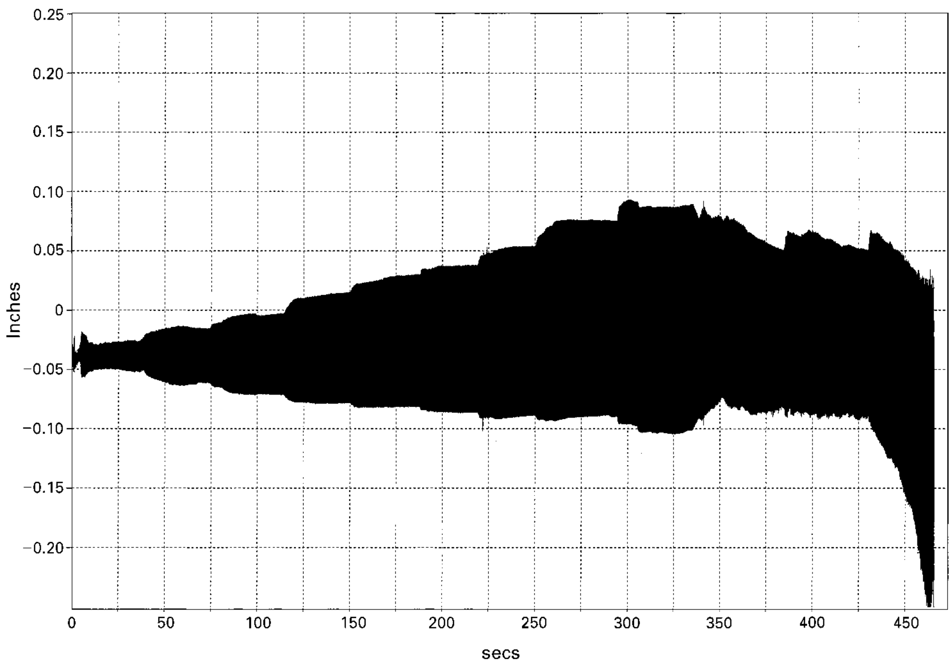

Figure 19. Second Koyna model: displacement at the top of the model.

the dam portion of the model, but rather appear to be a failure in the base of the model which acted as the foundation of the structure.

The conclusion from these data is that the material around the all-thread embedded in the base started failing at around the 330 s time frame and allowed the model to rock. As more material failed, the rocking increased resulting in the increasing vertical accelerations and decreased acceleration of the top initially. Eventually, the material failure around the all-thread was severe enough that the entire model could slide back and forth a small amount in the direction of the excitation. This is evidenced by the spikes in base acceleration shown starting around the 400 s time frame in Figure 16. This indeterminate boundary condition would be nearly impossible to model on a non-linear analysis time-step basis. In is believed that general comparisons can still be made based on the final accelerations and the material properties presented.

It was noted that after initiation of the crack that the top of the model began to slide before toppling occurred. The top portion toppled from the model approximately 1 s (14 cycles) after crack propagation.

CONCLUSIONS AND DISCUSSIONS

1. A new concrete mix design is proposed which shows promise for use in similitude testing. The mix uses bentonite to reduce strength properties of the concrete and can be readily adjusted to simulate various scales. The components may be mixed in mass and can be provided by commercial producers because no hazardous materials are used. Disposal is also easily accomplished by conventional methods.
2. The new mix produces strength and stiffness characteristics which nearly match the similitude requirements. More importantly, for non-linear modelling of the failure mechanism, the mix fails in a shear plane almost identical to conventional concrete.
3. The initially cracked model (model 1) and the monolithic model (model 2) showed general modal characteristics which were similar for small accelerations.
4. Model 1 is characterized as a kinematically non-linear model because it's initially cracked top section failed in a sliding mode. This model demonstrated that there was some initial bond on a typical shrinkage crack. This model showed that even a crack visible to the eye on multiple faces, must overcome some bonding before sliding can occur.
5. When sliding of a failed section initiates, the non-linear effect creates very large changes in the dynamic response under a constant sinusoidal input motion. The amplitude above the crack in this model actually becomes less than the base and the response is phase shifted. Put simply, the base can move back and forth beneath the top with the motion being only loosely coupled.
6. The monolithic model (model 2) failed with a material failure which was characteristic of previous models and is believed to be characteristic of cracks in actual cases.
7. During the monolithic test, a change in the base boundary condition created a highly non-linear and indeterminate boundary condition. This non-linear change also showed large changes in the dynamic response of the model which are easily seen in comparison to the constant motion input. Unfortunately, this same boundary condition change makes exact time history matching with numerical models impractical.
8. Both models failed at approximately $2.2g$ of acceleration. In the kinematic model (model 1), sliding created a slow progressive sliding during the cyclic motion. In the materially

non-linear model (model 2), a crack was initiated in less than $\frac{1}{30}$ of a second and sliding occurred for a number of cycles before the top of the model toppled. The toppling is inconsistent with previous models and is believed to be related to vertical accelerations produced by the boundary condition change.

9. Laboratory tests were performed on the material used to construct the shake table models to provide parameters typically needed in non-linear numerical models.
10. Results in the kinematic failure model (model 1, sliding) can conceivably be time step matched to verify non-linear models. Results from the materially nonlinear model (model 2) can be verified in a general manner to verify cracking pattern and acceleration required for failure.

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